1951

Summary report, Lehigh University, (1951)

Fritz Lab

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September 17, 1951

File No. 205

re: FUTURE PLANS

To Members of the Lehigh Project Subcommittee:

Gentlemen:

Our plans for the next year, on the basis of funds definitely available to the project on Welded Continuous Frames and their Components, include the following studies: (projects are described on Enclosure (1)).

1a. Continuing program of column research. (Mr. R. L. Ketter).
1b. Corner connection research. (Mr. A. Huber).
1c. Completion of research reports, analysis of data collected. (Dr. K. E. Knudsen).
1d. Portal Frame studies. (Dr. E. R. Johnston, Dr. K. E. Knudsen).

An additional program of research has been approved by the Office of Naval Research, although the work has only partially been financed.

2a. Inelastic Instability. (C. H. Yang).

Several proposals have been written since the last meeting on subjects that are either a part of the general investigation or are of interest to Committee members:

3a. Interior Beam-Column Connections.
3b. Collapse Strength of Columns (part of a monograph on the collapse strength of steel structures).

Among the programs of further research, or of continuing research, being considered but for which definite proposals have not been written are:

4a. Stress-strain Properties of steel.
4b. Residual Stresses in Columns.
4c. Additional Beam Investigation.

May we have your approval or criticism of the above program as further explained by the enclosures, in particular items 1a, 1b, and 1c which are recommended for the year beginning October 1, 1951.

Sincerely yours,

LSB:jb

Lynn S. Beedle
Project Director

Enclosures (2)

F O R E W O R D

It was planned to send several letters to members of the Lehigh Project Subcommittee in preparation for its regular semiannual meeting. Rather than send these out separately, the material has been gathered together under this cover. In addition to items essential to the meeting there are included certain new results of recent research that may be of interest. Some reference material has been added as indicated in the contents.

Progress Report No. 6 "Residual Stress and the Yield Strength of Steel Beams" is **not** discussed here since it has recently been distributed under separate cover.

-o-oOo-o-

This work has been carried out as a part of an investigation sponsored jointly by the Welding Research Council and the Department of the Navy with funds furnished by the following:

American Institute of Steel Construction
American Iron and Steel Institute
Column Research Council (Advisory)
Institute of Research, Lehigh University
Office of Naval Research (Contract No. 39303)
Bureau of Ships
Bureau of Yards and Docks

Fritz Engineering Laboratory
Department of Civil Engineering and Mechanics
Lehigh University
Bethlehem, Pennsylvania

September 17, 1951

Fritz Laboratory Report No. 205.13

DESCRIPTION OF PROGRAMS

1a. Column Program -- The test program is outlined in Progress Report K (November 20, 1950) and many of the tests have been completed.

A specific outline of future tests will be submitted separately for the specimens still available. The type of tests were presented generally in Progress Report K, page 17.

Several additional progress reports (stiffness, stress-distribution) are scheduled.

1b. Corner Connection Research -- Suggestions for further connection research are contained in a separate note Enclosure 2, dated July 13, 1951. Mr. Alfons Huber, presently a research fellow has returned to his home in Austria but expects to be back in the United States in January to carry out further studies on welded connections. As part of a course project, Mr. Huber studied built-up connections.

In the tentative proposal mentioned above, the first part of the work will be analytical, leading to a test program.

1c. Reports and Analyses -- We consider it important to make headway on a number of incomplete research reports for which most of the data has been collected. Subjects that must be reported on are: the ultimate strength of continuous beams, the influence of shear, and the deflection of continuous beams. Additional data has also been collected on "shakedown".

Dr. C. H. Yang may possibly have time to continue some of the above-mentioned reports. Dr. K. E. Knudsen who received his Ph.D. degree at Lehigh several years ago, returned in September and is participating actively on the project. The proposal of item 2a below also includes a consulting arrangement for Dr. Bruce G. Johnston.

1d. Frame Studies -- Dr. E. R. Johnston is working on the results of the two portal frames tests completed in February and in April. Our last two quarterly progress reports (3 April and 12 July) contained brief summaries of the tests. Dr. K. E. Knudsen is working on a short report of test apparatus.

A third frame test in which a haunched knee would be used remains in the program (Proposal of August 25, 1950). Depending on raising a small amount of additional money this test might be done during the coming year. Further experiments should also include combined vertical and side loading.

2a. Inelastic Instability (Local Buckling) -- Dr. Yang has been working on this project for the last few months. The proposal for this work is dated June 15, 1951. A Progress Report will be prepared leading to a program of beam tests in a range of sizes and shapes. A separate letter has been written discussing the local buckling problem.

3a. Interior Beam-Column Connections -- A fellowship on this subject has been proposed to the Welding Research Council (distributed by Mr. Spraragen under date of May 9, 1951). Numerous suggestions have been received which will be considerable value when the funds have been procured. Professor C. D. Jensen will direct this work, it being proposed that Mr. Edmund L. Kaminsky carry out the investigation.

In the event a fellowship program does not materialize in the near future, thought should be given to the relative priority of this work as related to the other programs.

3b. Collapse Strength of Columns -- It is understood that the proposal for this work is receiving favorable action by Sandia Corporation. No test work would be done. The work is related to a certain extent to the present column program, and is part of Dr. Bruce G. Johnston's monograph on the collapse strength of steel structures.

4a. Stress-strain Properties of Steel -- This subject is of direct interest to the Lehigh investigation and some work has been done during the past year as described in the quarterly progress reports. Further work is necessary but a proposal has not yet been written.

4b. Residual Stresses in Columns -- A preliminary proposal is prepared but is not yet ready for release. Some work has been done during the past year and a separate letter has been written outlining some of the principal results.

4c. Additional Beam Investigation -- At some later time a proposal will be written on the subject of beams with varying end restraints and on the influence of welding residual stresses that accompany the fabrication of beams by welding of a web to flange plates. The importance of both of these items stems from their influence on the deflection of the structure.

SUGGESTIONS FOR FURTHER CONNECTION RESEARCH

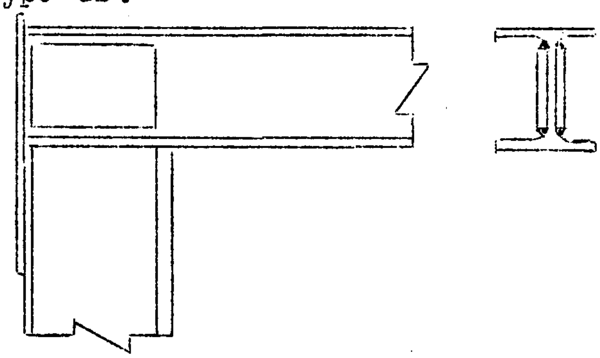
A. Straight Knees

(I) Repeat type 8B connection tests (connection L) but in a range of sizes. The theory developed in Part II predicted satisfactorily the rotations, but it is necessary to check this on a range of sizes. The 14WF30 is one suggested for this and is available at the laboratory. It is geometrically similar but nearly twice the depth. The influence of shape should also be examined experimentally.

(II) Test a connection similar to type 7 (probably use type 8C) except that additional plate material would be welded to the web to provide adequate thickness to carry shear force. Cost would be compared with the 8B type. Total thickness of web will be based on equations developed in P.R. # 4. Both the 8B13 and 14WF30 sections could be used. After an initial test, study the influence of changing the plate thickness. Plates would be welded to the fillets. The design is advantageous where a diagonal stiffener is objectionable.

(III) It will be noted that square knees were somewhat more flexible in the region of the theoretical elastic region than desirable for plastic structures. Since an economical knee type is desirable, one such approach over and above (II) is to test the type 1 connection (Fig. 5). An initial test is planned in a program at the University of Texas (*).

(IV) Following III, investigate the possibility of eliminating the vertical stiffening extension of the column flange, type 8D.



(V) Tests of one "satisfactory" connection type under two limiting conditions: pure moment and pure shear to study these limiting cases. This could be done experimentally as shown in Fig. 1.

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* See Proposal November 29, 1950.

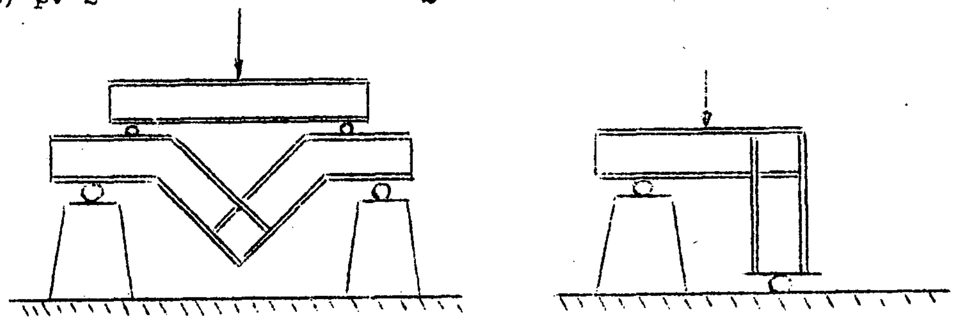


Fig. 1

(VI) Tests of "I" sections, using type 8 connections. The thicker webs will be more efficient in developing shear strength. A range of 2 or 3 additional sizes might well be studied.

(VII) Some tests of connections loaded in "tension" have already been completed and the study is underway.

B. Built-up Connections

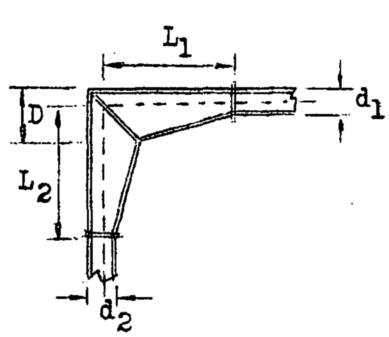
A considerable amount of analytical work remains to be done for this phase of the investigation. A portion of the analysis of tapered haunch connections (rotations and deflections) has been done, together with a study of stresses. These subjects will be reported on separately.

(I) All non-symmetrical haunches should be loaded unsymmetrically such that the moments are equal at the two haunch ends. This was not done in the case of connection C, for example.

(II) It was noted that the type 4 connections (D,E,F) do not represent an inordinate expense compared to the 8B type. It might be possible to improve the carrying capacity at little or no extra expense by decreasing the stiffening in type 4 knees. This is also being studied in the program at Texas.

(III) Further tests of the type 2B connection have been commenced at Lehigh, investigating the influence of increasing the length of loading arm, and the requirements for lateral support.

(IV) Additional analytical work is needed on the 2B connection. Referring to Fig. 2 some of the influences that should be investigated are,



(a) d/D (constant L)
(b) d/D (slope constant)
(c) L_1/L_2
(d) Influence of changing thickness of the inner flange.

Eventually, based on a given moment gradient, it is required to specify how to proportion the haunch most economically to develop the required strength. The possible further economy due to plastic hinge formation and the ability of this type to carry several loading conditions also requires critical study.

(V) The influence of shape of cross-section must be studied. However, a great deal of this is related to local buckling, a study of which is being carried out under a separate proposal.*

(VI) The influence of casing should be studied.

(VII) So far as curved knees are concerned, the rules of the AISC are evidently satisfactory in developing the full strength of the beams joined. For the larger radii of curvature, it might be worthwhile to study the improvement in carrying capacity as the radius is held constant but the flange thickness is increased.

(VIII) Testing completly built-up column and haunch assemblies would be of interest due to the rather large number of this type used at the present time. The transition from column to haunch to girder is so gradual that the limits of the haunch cannot be clearly defined. Personnel at the Bureau of Yards and Docks has expressed interest in such a program.

(IX) Connections which are so large that the proportions are no longer those of rolled sections will require special treatment. Development of elastic strength may be all that is possible or required.

C. Interior Connections

The problem of interior connections is treated in a proposed WRC fellowship program, distributed to the Structural Steel Committee.

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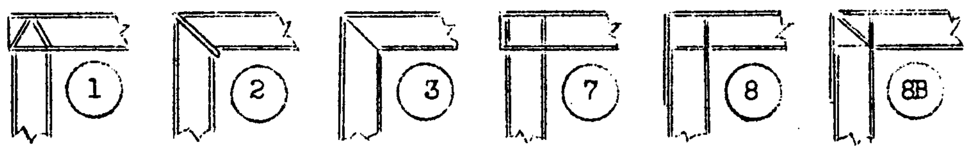
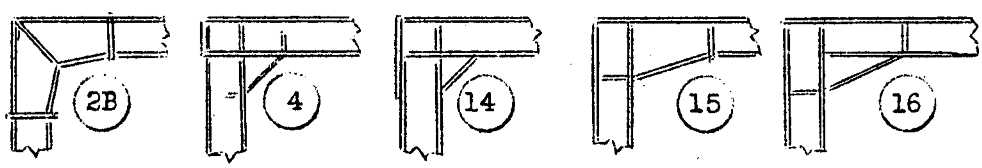
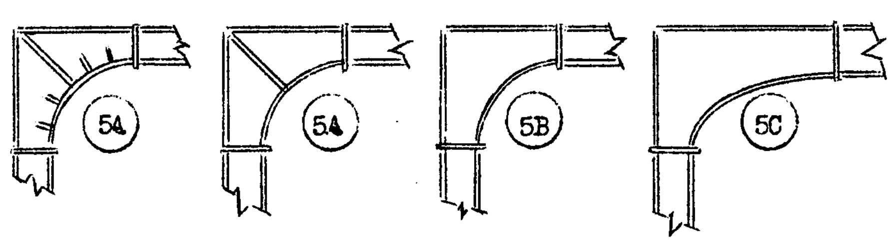
* January 15, 1951



Square Knees



Haunched Knees



Curved Knees

Fig. 5. PORTAL FRAME KNEES

September 15, 1951

Members Lehigh Project Subcommittee

Gentlemen:

Subject: Inelastic Instability (Local Buckling)

As an extension to our existing contract with the Office of Naval Research we submitted some time ago a proposal for a study of the "local buckling" problem as it applies to the flange elements of rolled shapes. An outline of the problem is attached as enclosure 1.

Beyond that contained in his disseration, Dr. Harry Yang has worked on the project for the past few months, but his report will not be complet prior to the September 24th meeting of the committee.

Enclosure 2, shows the central span (under pure moment) of two frames. The members for one of the frames were 8B13 sections, while an 8WF40 shape was used for the second. I should like to call your attention to the following points that illustrate the importance of the phononem of local buckling.

(1) Frame 1 (8WF40) carried over 25% more load than predicted by the simple plastic theory. At a central deflection of 20" (the space was 14' and the height 7') the frame was continueing to carry increased load in spite of the local buckling strain.

(2) Frame 2 (8B13) carried a load just equal to the predicted ultimate load. After a deflection of only a few inches the frame collapsed following the severe local buckling shown in the photograph.

(3) The flange width $\cong$ thickness radius of the two sections are identical (b/t = 16)

(4) The increased curvature developed in frame 1 (8WF40) over the 8B13 shape is shown in the photograph.

(5) As evidenced by the difference in deflections at the end of the test, frame 1 (8WF40) had a substantially longer plastic range than frame 2. In considering the collapse strength of steel structures from the point of view of bomb damage, the 8WF40 shape possess very definite advantages over the 8B15.

Sincerely yours,

CAY/ WB

Enc. 2
LSB/oh

Harry Yang
Lynn S. Beedle

I N E L A S T I C   I N S T A B I L I T Y

Lehigh University

Excerpt from SUPPLEMENTARY PROPOSAL
(January 15, 1951)

SCOPE

It was indicated in the continuous beam test program supported by the Department of the Navy (ONR, BU Y & D, and BuShips) that some rolled sections of Wide Flange shape will not develop their plastic hinge value due to local buckling of the compression flange. Since plastic analysis is realistic only if plastic "hinges" are properly developed, it is important at this time to make a systematic study of the problem of local buckling of the compression flanges of "I" and "WF" shapes. It will influence both the analysis for maximum load-carrying capacity of existing structures and the design of new structures. Particularly at the present time, when it is necessary to design structures to make use of the least amount of strategic material, the possibilities of "plastic design" become attractive.

The ultimate aim of this investigation is to establish a specification for the required geometric proportion of I and WF shapes so that plastic hinges can be developed and maintained through a considerable range of rotation under various loading conditions without reduction due to local buckling. Attention will be given to the following aspects of the problem:

(a) Designation of the geometric proportion of those avaibable sections which meet the requirements of strength and rotation capacity.

(b) Recommendation for possible modification of presently available shapes.

The former would receive major emphasis.

This program will start with an analytical investigation of the instability characteristics of short compression members of structural steel in the plastic and strain-hardening range. It will lead to a study of simply-supported beams under two extreme loading conditions.

Tests are needed and will be proposed after literature review and analytical studies.

A FURTHER COMMENT ON THE INELASTIC INSTABILITY PROGRAM

During recent years there has been concentrated research into the inelastic behavior of steel structures. It is now apparent that some of the new theories formulated may be applied to practical design thus bringing about an economy of material. However, the analysis of the ultimate strength of continuous structures, based on the simply plastic theory, is realistic only if plastic hinges are properly developed.

The results of our present research program (Welded Continuous Frames and Their Components) has revealed that structural shapes such as those used in connections, beams and columns, may not develop their full plastic hinge strength due to inelastic buckling of the compression flanges.

A large amount of the previous work done in inelastic buckling of structural members is confined to the light alloys which exhibit a non-linear stress and strain relation. The case of perfect plasticity has scarcely been discussed.

Structural members of perfectly plastic material theoretically have no resistance to buckling when the average compressive stress has reached the yield point, no matter whether the buckling concept based on the tangent modulus or the double modulus is used. Thus, the compression flange elements of structural elements should buckle when the compressive yield stress is reached through the flange thickness. As evidenced by large numbers of tests, such buckling does not necessarily occur in all rolled shapes, and in some cases further deformation must be applied to the members before such buckling is observed.

The usual coupon compression tests of structural steel specimens have shown a greater resistance to buckling which is probably attributed to the yielding process of steel. This will be discussed in further detail in a forthcoming progress report.

To establish an effective value equivalent to the tangent modulus for steel for calculating the buckling strength of small compression steel members in the plastic range would require a large number of compression tests of precisely aligned steel members which would be strained through the plastic range and into the strain hardening range. Results will be correlated with the analysis of the buckling strength of steel plates and lateral buckling strength of beams.

The ultimate aim of this investigation is to establish a recommendation for the required geometric proportion of I and WF shapes so that plastic hinges can be developed and maintained through a considerable range of rotation under various loading conditions without reduction due to local buckling. New specifications for the lateral support of steel structures in plastic design would be introduced after proper methods of predicting the lateral buckling strength of beams were formulated.

This 6-month program is starting with a search and review of the available literature. Studies will then be made on analytic solutions of the buckling problem of steel elements in plastic range. Thereafter a program of experimental study will be recommended.

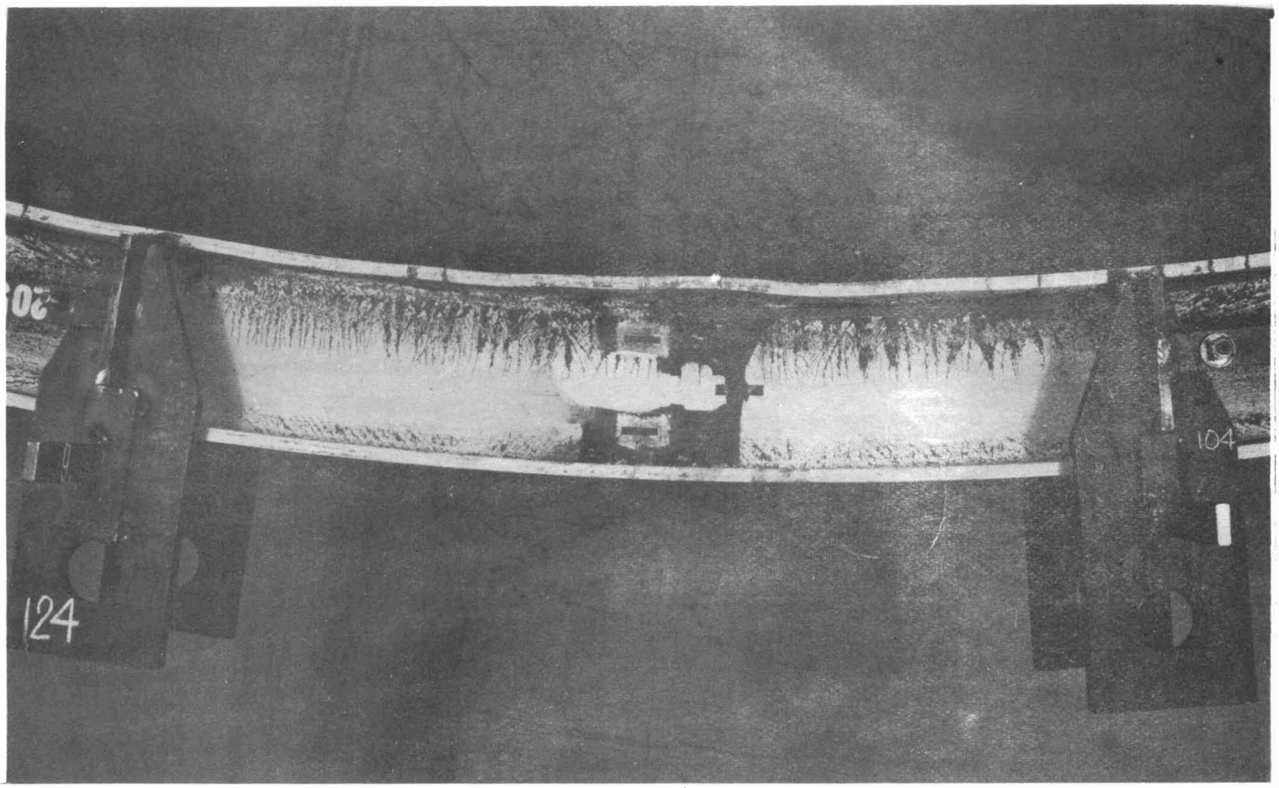


Fig. 1 - Central portion of Frame 1 under uniform moment. 8WF40 shape.

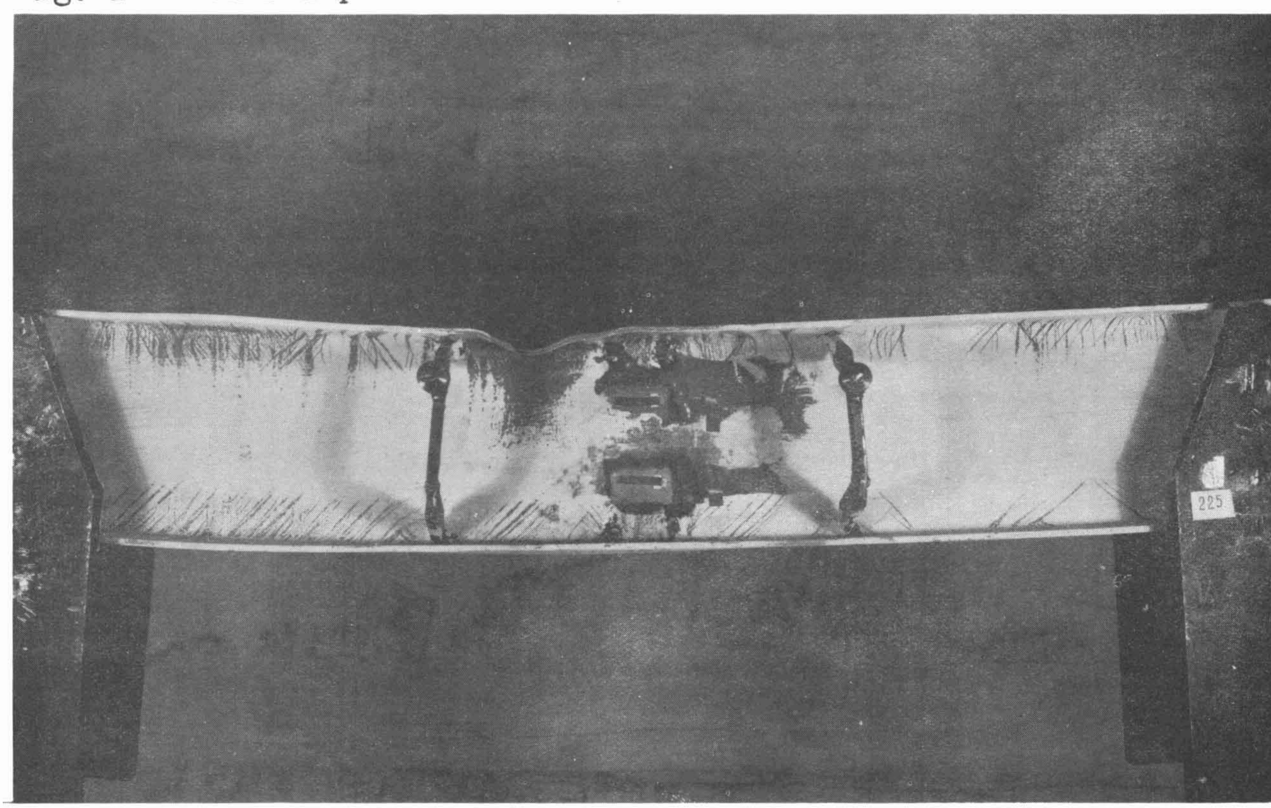


Fig. 2 - Central portion of Frame 2. 8B13 shape.

TO: Members, Lehigh Project Subcommittee

SUBJECT: Column Test Results

Gentlemen:

Dr. Bruce G. Johnston was asked by the Column Research Council to represent it at the October, 1951, meeting of the Structural Engineers Association of California (to be held, incidentally, at Yosemite National Park). He will present a survey of the work of the Council. In answer to his request we have furnished him with a few paragraphs describing our column investigation.

The following material is a slightly revised version of what was sent to Dr. Johnston. It is being forwarded to you since it contains some new test results. The introductory statements have been omitted.

Lehigh Project: COLUMNS IN CONTINUOUS FRAMES

The three-dimensional interaction curve shown in Fig. 1 has been developed to provide a basic framework for discussing the theoretical and experimental work. For axially-loaded members with no applied end moments, the theoretical curve is the familiar column curve of load (or stress) plotted against slenderness ratio. The theoretical curve shown is based upon idealized structural steel. For members without axial load, the curves in the moment-length plane are beam curves. As the length increases, lateral buckling becomes a more serious condition.

Between these two limits, curves in space define two interaction conditions:

(a) Yield strength (M_{yc})

(b) Collapse strength (M_{pc})

For the former it is seen that there is a range for the particular loading condition shown in which the yield strength is theoretically unaffected by slenderness ratio. If generally confirmed by test, then for a particular range it would be unnecessary to apply correction to F_a and F_b. For comparison, the surface formed by the AISC interaction formula is shown by the shaded portions.

Equations have been developed and summarized [1] for predicting the yield strength for each of the loading conditions being used in the program. To a more limited extent methods have been outlined for predicting the collapse strength.

The results of numerous tests on 8WF31 columns are shown in Fig. 2. This Figure amounts to a "cross-curve" of the previous figure at L/r = 56. Tests 3 and 4 are in reasonable agreement with the theory. But T5 collapsed at a load less than the predicted initial yield load.

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[1] Progress Report L.

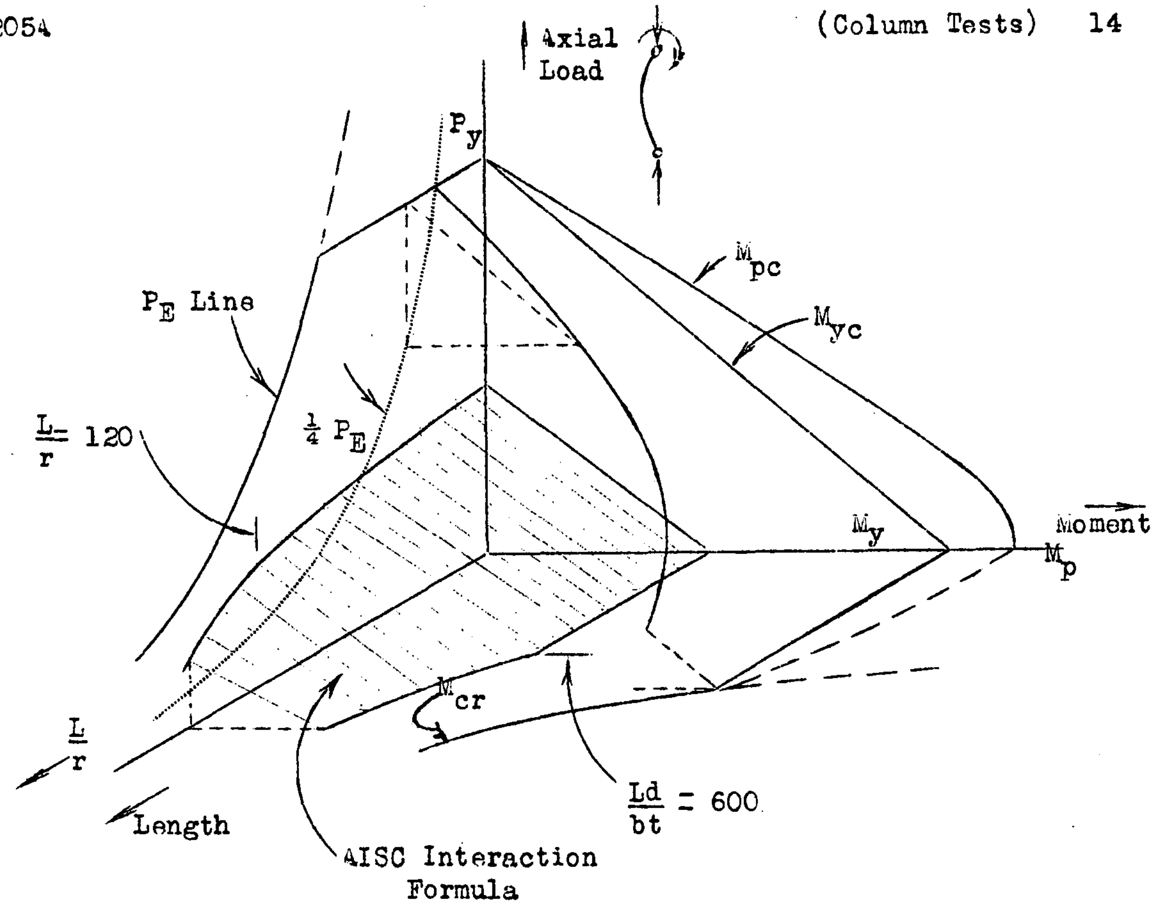


Fig. 1

The results of numerous tests on 8WF31 columns are shown in Fig. 2. This figure amounts to a "cross-curve" of the previous figure at $L/r = 56$. Tests 3 and 4 are in reasonable agreement with the theory. But T5 collapsed at a load less than the predicted initial yield load.

The short cross-lines indicate loads at which "yield lines" were observed. They indicate that a residual stress of about 12 ksi was present. In the case of T5, this early yielding allowed the member to buckle sidewise. For comparison the allowable working loads according to the AISC formula would fall along the dot-dash line.

The other tests at low P/Pcr values are for other load conditions in which the moment along the column is also a maximum at the ends.

The results of column tests using the same slenderness ratio as in Fig 2 are shown in Fig 3. However, under the "single curvature" load condition the moment along the column is a maximum at the center. If, due to residual stress, yielding occurs at a lower load than predicted, failur occurs by lateral-torsional buckling. As shown by the tests, except at a very low value of P/P_{cr}, the failure load is considerably less than the predicted value of initial yield load.

This investigation in the Fritz Laboratory at Lehigh University is still underway. However, the tests carried out up to the present time indicate:

(a) There is a range of loading conditions in which steel columns will carry more load than predicted and another range in which columns collapse below the predicted yield strength. Since most column formulas are based upon stress as a criterion, this indicates that on the one hand the formulas are too safe and on the other hand they may not be over conservative.

(b) Columns loaded in single curvature do not develop their yield strength unless the axial load is relatively low.

(c) The influence of residual stress is more important than previously realized in reducing the strength of these columns bent about their strong axes. Yielding at a cross-section markedly reduces lateral buckling strength.

(d) For load conditions other than single curvature the influence of axial load may in some cases be ignored at low axial loads (f_a/f_b about 10%).

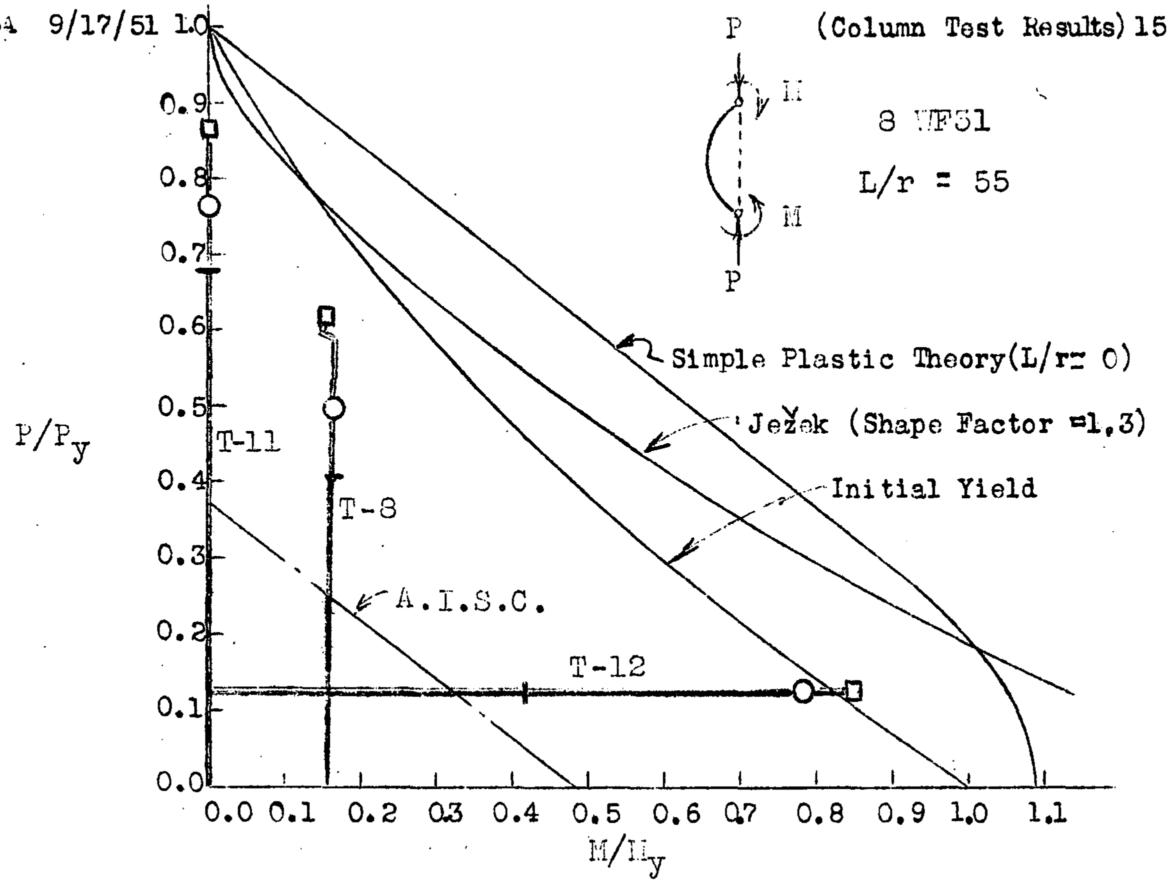
(e) After more analysis and test work, it may be possible to specify a range of L/r and P/P_{cr} in which the influence of L/r may be neglected.

(f) An analytical solution to the inelastic lateral buckling problem is an important step in further research.

Sincerely yours.

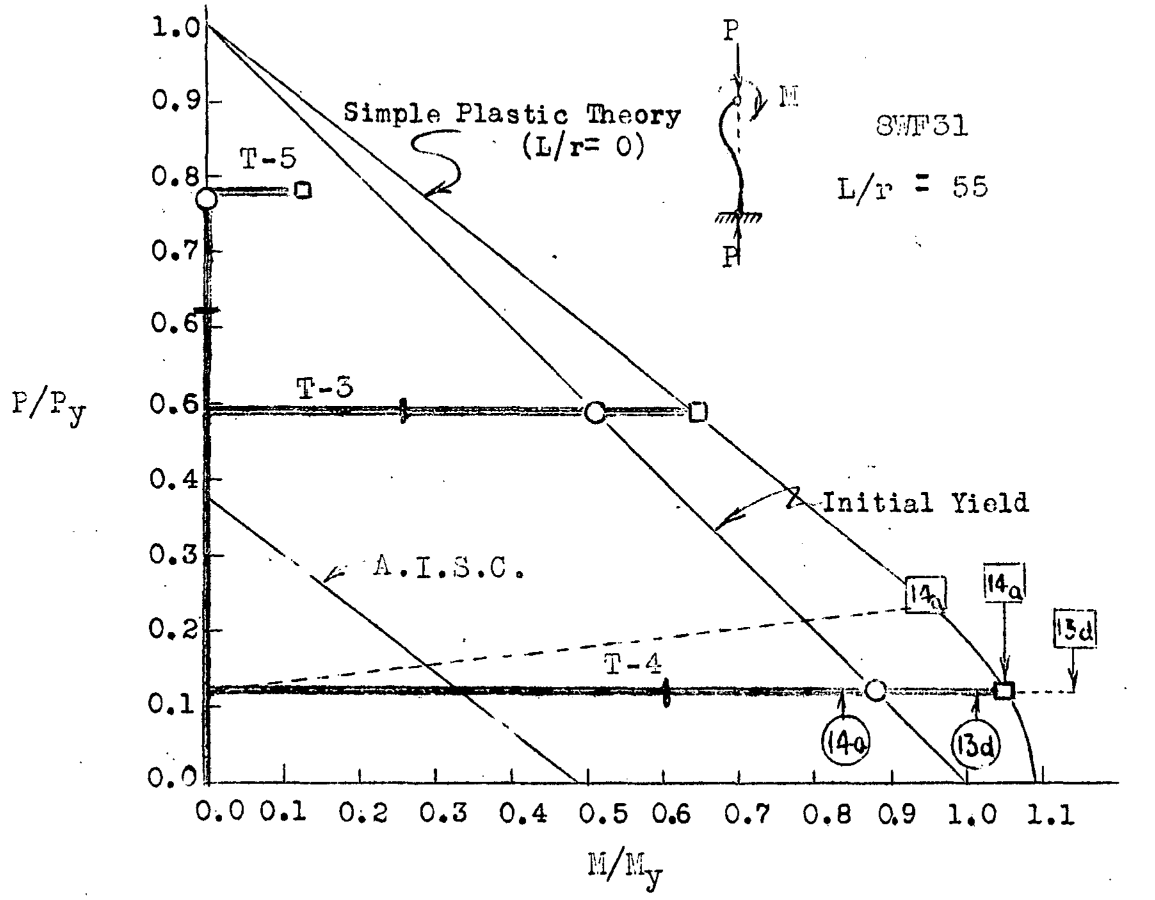
Robert L. Ketter

Lynn S. Beedle



8 WF31
L/r = 55
P
M
Simple Plastic Theory(L/r= 0)
Ježek (Shape Factor =1.3)
Initial Yield
A.I.S.C.
T-11
T-8
T-12
P/P_y
M/M_y

Fig. 3



8WF31
L/r = 55
P
M
Simple Plastic Theory (L/r= 0)
Initial Yield
A.I.S.C.
T-5
T-3
T-4
14a
13d
P/P_y
M/M_y

Fig. 2

17 September 1951

File No. 220A

TO: MEMBERS, LEHIGH PROJECT SUBCOMMITTEE

RESIDUAL STRESSES IN COLUMNS

Gentlemen:

You have undoubtedly noticed in the recent Quarterly Progress Reports that we have often mentioned the subject of residual stresses. This stems from the fact that early in the experimental work yielding was observed in the flanges of WF sections at lower loads than predicted on the basis of coupon tests. The Lehigh Project Subcommittee recommended that residual stresses be measured early in the program and some results were presented in the first progress report of the project (Welding Journal, November 1948).

Since 1949, Dr. Bruce Johnston has been emphasizing the importance of residual stresses in reducing the critical load in columns. Then a column test was carried out (part of the approved program) showing a reduction in strength over that predicted on the basis of small coupon tests. The problem has also been treated theoretically in Progress Report 6. As part of a course project, Mr. Huber then carried out a pilot program in which he tested a 20" length of an 8WF31 member with flat ends. Hereafter this is called the "residual stress specimen". An average stress-strain diagram was obtained which was used to compute a column curve according to the tangent modulus concept.

It is the purpose of this letter to show the correlation between the results of pin-ended column tests and the theoretical column curve based on the above average stress-strain diagram. A preliminary report has been prepared and will be distributed after further editing.

In Fig. a is shown the stress-strain diagram as determined by strain measurements on a small coupon as compared with the average stress-strain curve for the full 8WF31 cross-section containing residual stress. Measurement for the latter were made with a simple dial gage technique, readings being taken at each of the four corners over a gage length equal to that of the specimen.

In Fig. b are shown column curves determined from the tangent modulus of the curves shown in Fig. a. Plotted upon the curve are the critical loads of three tests at slenderness ratios of approximately 28, 42 and 56.

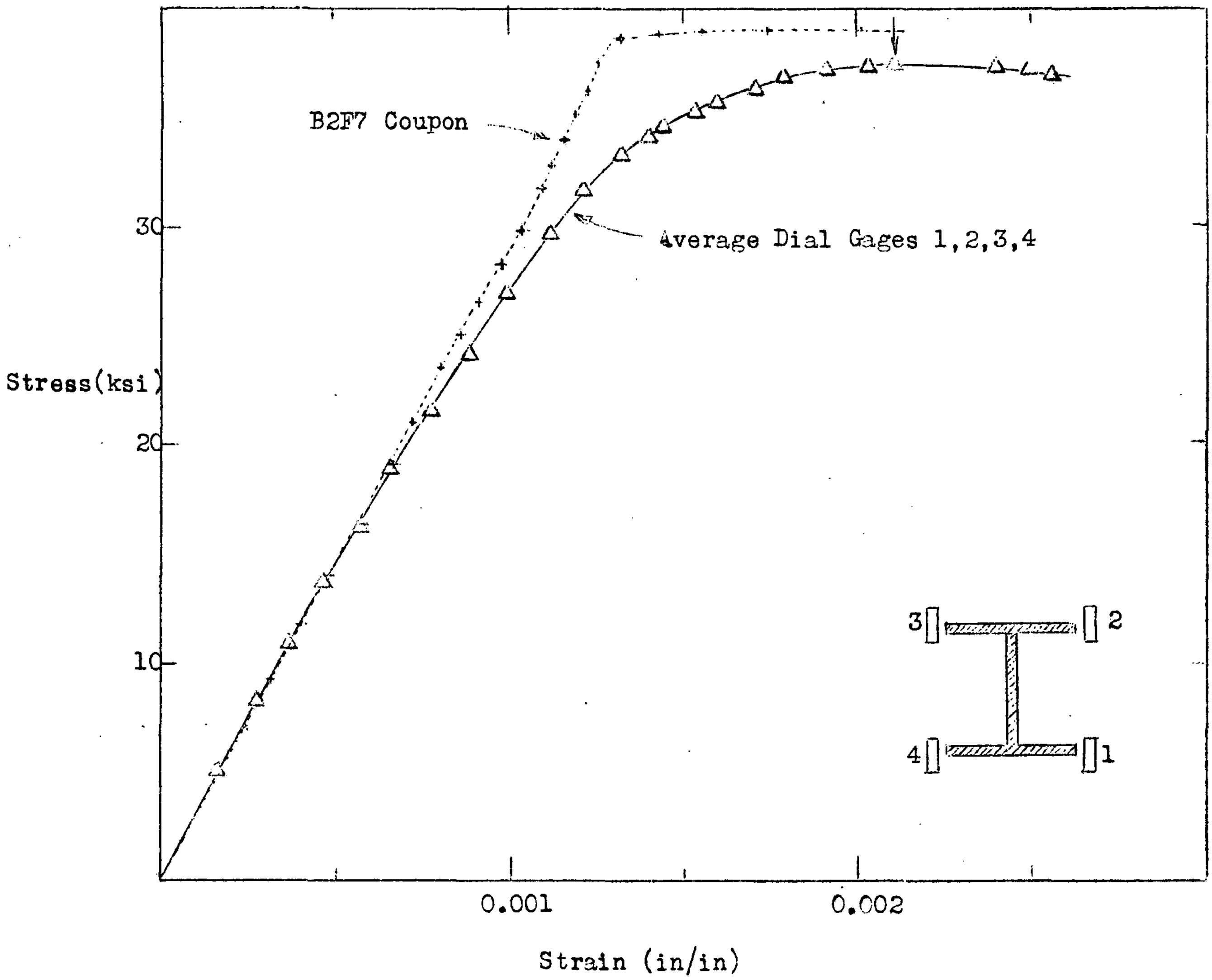


Fig. a

This study of a short compression specimen resulting in the curve of Fig. b is only a pilot investigation. However present evidence indicates the following:

(1) A satisfactory column curve cannot be obtained on the basis of tests of small samples cut from various places in the cross-section.

(2) From Fig. b, as expected, the percentage reduction is greatest in the range L/r = 90. No tests fell in this range and they should be done.

(3) Good agreement with the column curve is obtained for T11 and T18. T15 carries less load than predicted on the basis of the average curve determined from the "residual stress specimen".

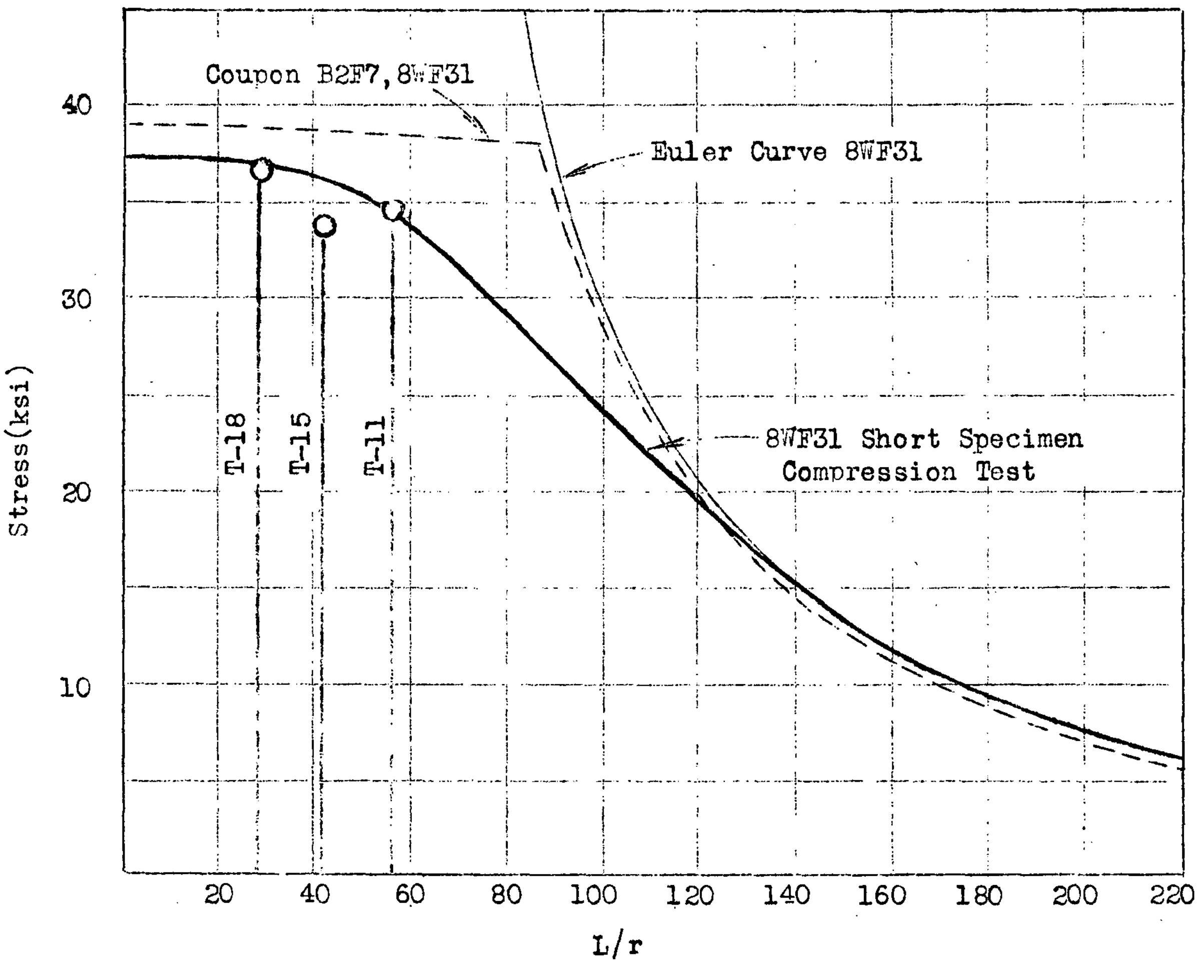


Fig. b

~~In Fig. b are shown column curves determined from the tangent modulus of the curves shown in Fig. a. Plotted upon the curve are the critical loads of three tests at slenderness ratios of approximately 28, 42 and 56.~~

(4) The column specimens came from two ingots of steel from the same heat. T11 came from one rolling ("A" material) and T15, T18 and the residual stress specimen came from the second rolling ("B2" material). The residual stress level has not been measured as yet for the "A" material.

(5) The three columns should theoretically buckle about the "strong" axis of the section by virtue of the "fixed end" condition against bending about the weak axis. In each case, however, final failure was by combined bending and twist.

(6) A theoretical curve could be drawn in Fig. b for collapse on the basis of the "effective I" concept discussed in Progress Report 6, section VII. This curve would be below the column curve determined from the "residual stress specimen".

Through arrangements with the Bethlehem Steel Company the project staff visited their plant and observed the rolling, cooling, and cold-straightening of 8WF31 specimens (the same shape as used in the column test program). Other shapes were examined after cold-straightening. The purpose of the visit was to determine at what stage in the process the "yield lines" were formed, since nearly every steel member delivered shows evidence of prior plastic strain. The following comments are made as a result of this examination:

(1) Almost without exception every steel member is cold-straightened in a process that consumes a considerable amount of time.

(2) For the sections observed <u>there are no "yield lines" present in the specimens after cooling and prior to cold-straightening</u>. During the "gagging" process, the formation of local plastic yield zones could clearly be seen (Examples of "yield lines" are shown in Progress Report 6, Figs. 30 and 31).

(3) There is plastic deformation of the flange center during the cooling process, a process which is basically responsible for the formation of cooling residual stresses. However, the mill scale in the region contains no yield lines, although it often has a rough appearance.

(4) In making residual stress measurements in rolled specimens one should note carefully the location of "yield lines" in the mill scale, since they are evidence of a cold-straightening pattern. Cooling residual strain measurements should of course be made where

there are no local yield zones, else erroneous conclusions as to the yield stress level may be drawn.

(5) Coupons for material property tests should also be selected taking care to note the cold-bending yield lines. Specimens which have been cold-worked will give a different result than those which have not and it is tentatively suggested that specimens be taken from portions which have not been cold-straightened for more representative indication of the strength of a member.

(6) There will certainly be variation in residual stress along a member and across the section. However it is expected that the cooling residual stress pattern will be rather uniform and that the greatest variations will be due to the local areas of cold-bending.

Research Committee C of the Column Research Council has recommended that research on residual stress be carried out and we hope to be able to participate in this work. Among the early investigations that are worthwhile is a study of the residual stress as influenced by shape of cross-section. A tentative chart organizing the over all residual stress problem has been prepared for criticism and follows.

RESIDUAL STRESSES IN COLUMNS

I. Mechanism for the Development of Residual Stress

II. Magnitude and Distribution of Residual Stress
   - A. Type of Material
   - B. Cross-sectional Form
   - C. Method of Fabrication (rolling,welding)
   - D. Variation along a Member
   - E. Variations through the Thickness

III. Influence of Particular Residual Stress Patterns on Column Strength
   - A. Type of Material
   - B. Cross-sectional Form
   - C. End Restraint
   - D. Loading (including combined bending and axial load)
   - E. Variation with L/r

IV. Technique - How to Develop Column-Strength Curves for Materials with Residual Stress
   - A. Measurement of Residual Stress
   - B. Testing full Cross-section
   - C. Comparison of "Coupon" Tests with Cross-section Tests.

Sincerely yours,

Lynn S. Beedle
For The Project Staff

LSB:fs

September 17, 1951

To: Members, Lehigh Project Subcommittee

Re: Evaluation of Plastic Design

Gentlemen:

Some of you have undoubtedly received copies of recent correspondence on a "discussion" of the general objectives and rational basis of our project at Lehigh. This series of letters was initiated by a proposal for research on interior connections and was extended into more general areas by a letter from Mr. F. H. Dill of the American Bridge Company.

As a result of the exchange of correspondence, Mr. Spraragen has suggested that a separate paper be published which would describe our objectives, summarizing the important points contained in the letter plus other related material. Such a paper is to be prepared and will be sent to members of the Subcommittee for their criticism.

Since some of the letters have received separately rather general distribution through Dr. Bruce Johnston, it was thought that you would like to be informed. Consequently, excerpts of three letters are attached. We will be glad to receive your suggestions. The three letters quoted from are those of Mr. F. H. Dill (May 17, 1951), Dr. Bruce Johnston (May 29) and a letter of mine (August 14). I have omitted parts not related to the evaluation of plastic design.

Sincerely yours,

Lynn S. Beedle
Project Director

LSB:jb

Enclosure (1)

Excerpt of letter from F. H. Dill to W. Spraragen, May 17, 1951

"The idea of extending the design of structures to include dependence on plastic action of primary members has been pushed by some of the Lehigh University staff as a pet project for ten years or more. Their whole program flies in the face of the knowledge that structural steel under many common circumstances has no plastic action. Even when there is plastic action, the stress required to initiate it may be unpredictably 25 per cent greater than that which is usually assumed and the ultimate structural behavior can be quite different from that supposed in the design. Structural engineers who recognize these facts can hardly be expected to design structures on the assumption of plastic action of primary members."

Excerpt of letter from Bruce G. Johnston to W. Spraragen, May 29, 1951

"My main purpose in writing to you (again) is to completely refute the statement in the fourth paragraph of Mr. Dill's letter:

> "The idea of extending the design of structures to include dependence on plastic action of primary members has been pushed by some of the Lehigh University staff as a pet project for ten years or more."

I have certainly been familiar with the aims of the Lehigh work during the past ten years and hope that this letter will correct any misunderstanding concerning them.

The Lehigh Project on "Continuous Welded Frames and their Components" was not originated by Lehigh, but was suggested by the Structural Steel Committee of WRC. In suggesting the program, one of the members of the Committee stated"...to exploit the advantages of welding, further research should be concerned with fully continuous beam and frame structures." We were glad to proceed with such a program and our primary aim has been simply to learn all that we could about the behavior of welded frames and their components both in the elastic and plastic ranges, not to advocate a particular design procedure. One of our principal aims has been to explore the limitations of using the plastic range in design of continuous frames. The most obvious limitations are the effects of repeated load, local buckling, and the possible brittle failure during low temperature. In talking with Professor Baker, the proponent of plastic design in Great

Britain, during his visit to Brown University some years ago, I argued that the increased load capacity sometimes available through plastic action of frames should be taken advantage of only so long as the actual allowable working stress remained within the elastic range. I also discussed this idea with Mr. Higgins and found that he had been thinking along the same line. In other words, I have never advocated and do not now advocate average allowable working stresses in primary members that exceed the elastic limit. In other words, plastic strength should be used as a basis of reserve factor of safety.

If the foregoing idea is not sound, then much of current specification practice is also not sound. We would have to abandon the very high allowable average stresses due to bending, shear, and bearing that are used in the design of pins, rivets, and local points of contact in all current structural design specifications. Actually, one could demonstrate that the yield point is exceeded in local regions of many existing steel structures. We simply do not calculate the actual maximum stress and therefore we already depend upon plastic action to insure the safety of our structures, since average stresses form the basis for design. Unfortunately, this procedure does not always give good results, as is demonstrated by the recent failure of the continuous welded bridge in Canada and other similar failures of structures both in this country and abroad.

I must admit that one of our reports, Progress Report 3, "Plastic Design and the Deformation of Structures" tends to refute my argument. Nevertheless, this report has as its main argument the contention that the true criterion of safe load is deformation - certainly a limit on the blind use of ultimate plastic strength as a basis for design. The final statement in this report points out that the working loads will usually be in the elastic range. I think they should always be in the elastic range except for wartime temporary structures or for very infrequent overloads such as wind loads of hurricane force.

Actually, I do not believe that we have any great difference of opinion with Mr. Dill. He has come to the conclusion that because we were studying plastic behavior, we also advocated the use of permitting plastic action at working loads. This is simply not true, at least as far as my own thinking has been concerned. In the case of railway and highway bridges, plastic action cannot even be used to any great extent in modifying permissible working stresses because the real controlling factor is fatigue failure rather than failure under static load.

There is one very important application of what we are learning about ultimate strength of continuous structures, namely, the prediction of the ultimate strength during a possible atomic blast. I feel that the work of Lehigh is more than justified simply by this aspect alone even though this may not have been a primary cause for its initial undertaking."

Excerpt of letter from Lynn Beedle to ~~W. Spraragen~~ Mr. Dill, August 14, 1951

"What I have to say in the following is not a defense of "Plastic Design" because our first purpose is to determine the facts about structural behavior. We would be the first to say that more work needs to be done before general conclusions can be drawn. Rather, this letter is a set of comments which seems pertinent to the discussion that has been under way.

(1) I believe in one of your lettersthe statement was made that "none has been enough convinced of the certain presence of plastic strength to place dependence on it in the design of structures". When in England in 1949 I found it quite interesting to learn that the British Standard Specification, BSS449, now allows the designer to use the "load factor" design method so long as due account is taken of deformations and that accurate methods of analysis are used. Probably this is based on work done by Baker and others in England.

This does not mean that we endorse this specification. As far as I know, buildings have not been designed, even in England, by the so called "plastic method".* However, certain shelter-type structures have been designed by these methods to carry one-cycle overloads by plastic deformation, and in actual wartime conditions appear to have performed remarkably well.

(2) I wish to emphasize and support Dr. Bruce Johnston's statement that we do not advocate a particular design procedure. One of the principal aims of the investigation has been to explore the limitations that may be involved in the utilization of the plastic range in design.

(3) To be sure in many structures stress is a limiting factor. Design for fatigue loading must certainly be based on a limiting stress. In materials which fail by brittle

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* Since writing this letter, Mr. Higgins has called our attention to an article in a recent issue of Transactions, Institute of Welding (June 1951) which discusses the construction of a new structure in England designed by plastic methods.

fracture stress must be the limitation and basis for design. Generally in conventional design the limiting stress is the yield point divided by the factor of safety. However, in the presence of the above mentioned factors further reductions are necessary. Naturally, the usefulness of plasticity in structural design is certainly limited by the same factors when they are present.

However, there are certain classes of continuous structures in which stress as such is not a limiting factor. Except for a few cycles, fatigue may not be involved. Brittle fracture of rolled wide-flange and I-shaped sections of 8" and 14" depths simply has not been observed in our laboratory. As you undoubtedly know, a program of impact tests has been underway at Columbia in which both butt-welded and as-delivered beams were tested under static load and under impact. The as-delivered beams, even at low temperature (-40°F.), deformed beyond usefulness before any fractures occurred.

Thus, for the class of structures in which fatigue, buckling, or brittle fracture is not a primary problem, the real limitation is deflection. To whatever extent the deflection may be tolerated, the corresponding load may be used as a basis for design.

(4) Our recent paper in the Welding Journal, "Plastic Design and the Deformation of Structures", to which Dr. Johnston referred, pointed out that at the working load the stress in the particular structures examined would be in the so-called elastic range. This conclusion was obtained solely on the design basis of limiting the deflection so that, at the full load,* the deformations were still "controlled". Applied to one example in the paper, that of a fixed-ended beam, the structure would be "plastic" at the full load. Applying the same philosophy to a simply supported beam, the structure would be entirely elastic both at the working and at the full load because the elastic limit of such a member (assuming WF sections) is identical with the load at which deflections increase rapidly with further increase in load.

The philosophy might be summarized by stating that structures which have "plasticity" and in which deformation under load is not limited by temperature, fatigue, etc., should be designed on the basis of limiting the deflection rather than upon limiting the stress. In such a case the attainment of a particular stress level at one cross section is not accomplished by failure of "collapse".

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* Full load is defined as the working load multiplied by the factor of safety.

(5) In statically determinate structures such as simply supported beams, there is no justification in the use of plastic design methods. One reason is that the theoretical yield point load corresponds to the load at which deflections commence increasing at a rate that is much more rapid than at lower loads. Plastic and elastic methods give the same answer....also pointed out in the above-mentioned paper.... because the basis for the selection of "full load" in the report is based on the load at which deflections become uncontrolled.

(6) In tests of structures it is not at all unusual to observe local yielding at loads as low as 25% of the predicted initial yield load. This is usually at points of stress-concentration or in the vicinity of a connection where there are also residual stresses. Thus, at present-day working loads, most structures have a permanent set of undetermined magnitude. If one were to attempt to design on the basis that no yielding were to be allowed, then the factor of safety would have to be raised. Engineering experience demonstrates that this is not necessary.

(7) Referring to your letter of June 12, (the first paragraph on the top of Page 2), the demonstration that "there is always a predictable amount of plastic action or reserve plastic strength in structural steel" is impossible for all types of structures for reasons that have been mentioned earlier in this letter. However, no one design method can be applied to every type of structure. Such a practice would certainly be wasteful of materials and Dr. Johnston's example of the higher allowable working stresses in pins illustrates this. It is our understanding that plastic design could only be used when the structure could deform plastically as assumed. Thus, the engineer will have the best design for supporting given loads who uses the least amount of material consistent with safety and the design requirements, and who is able to arrive at the design with the least overall effort. For certain structures use of the so-called plastic method might give the best answer. However, the primary purpose of our research is to study welded continuous structures, to determine their behavior in the elastic and plastic ranges, and explore limitations in the application of plastic range design.

LIST OF REPORTS

Published Progress Reports

1. Luxion & Johnston--PLASTIC BEHAVIOR OF WIDE FLANGE BEAMS
Welding Journal, Nov. 1948, p. 538-s.

2. Beedle, Ready, & Johnston--TESTS OF COLUMNS UNDER COMBINED THRUST AND MOMENT
SESA Proc., Vol. VIII, No. 1, Dec. 1950, p. 109.

3. Yang, Beedle & Johnston--PLASTIC DESIGN AND THE DEFORMATION OF STRUCTURES
Welding Journal, July 1951, p. 348-s.

4. Topractsoglou, Beedle & Johnston--CONNECTIONS FOR WELDED CONTINUOUS PORTAL FRAMES
Part I: TEST RESULTS AND REQUIREMENTS FOR CONNECTIONS
Welding Journal, July, 1951, p. 359-s. (ONR Technical Report No. 1).
Part II: THEORETICAL ANALYSIS OF STRAIGHT KNEES
Welding Journal, Aug. 1951, p. 397-s. (ONR Technical Report No. 3).
Part III*: DISCUSSION OF TEST RESULTS AND CONCLUSIONS

5.* Ketter, Beedle & Johnston--COLUMN STRENGTH UNDER COMBINED BENDING AND THRUST

6.* Yang, Beedle & Johnston--RESIDUAL STRESS AND THE YIELD STRENGTH OF STEEL BEAMS
Mimeographed Sept. 10, 1951.

Progress Reports Not for Publication

A Beedle, Ruzek & Johnston--PLANS FOR CONNECTION AND COLUMN TESTS
Nov. 26, 1948

B Yang--PLASTIC BEHAVIOR OF CONTINUOUS BEAMS
May 26, 1949

C Chen--STRENGTH OF COLUMNS UNDER COMBINED BENDING AND COMPRESSION
May 27, 1949

D Ruzek & Topractsoglou--TEST OF A RIGID FRAME KNEE
June 1, 1949

E Topractsoglou, Ruzek & Beedle--WORKING DRAWINGS FOR THREE CONNECTION TESTS. PROPOSAL FOR ADDITIONAL TESTS
June 1, 1949

F Beedle--GENERAL SUMMARY REPORT
July 19, 1949

G Beedle--STRUCTURAL RESEARCH AT CAMBRIDGE UNIVERSITY
Jan. 30, 1950

H Beedle & Yang--DISCUSSION OF "FLEXURE OF I-SECTIONS ABOVE PLASTIC RANGE" BY W.H. WEISKOPF
Feb. 20, 1950

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*Publication scheduled in Welding Journal.

I 205 Staff--SPECIAL SEMI-ANNUAL REPORT TO SUBCOMMITTEE
March 8, 1950

J 205 Staff--ANNUAL REPORT
Jan. 15, 1951

K Ketter & Beedle--SOME RESULTS OF COLUMN TESTS, PROPOSED PROGRAM
Nov. 20, 1950
(ONR Technical Report No. 4).

L Ketter & Beedle--INTERACTION CURVES FOR COLUMNS
April 20, 1951
(ONR Technical Report No. 2)

<u>Reports Available on Loan:</u>

205C.5 Topractsoglou--CONNECTIONS FOR WELDED RIGID PORTAL FRAMES
Vol. I and II.
Dissertation, Lehigh University, 1950

205B.3 Yang--THE PLASTIC BEHAVIOR OF CONTINUOUS BEAMS
Dissertation, Lehigh University, 1951

Chen--on the subject of ELASTIC LATERAL BUCKLING OF I-SECTION COLUMNS
Dissertation, Lehigh University, 1950

205.07 Beedle--PLASTICITY IN STRUCTURES
Notes on Research at Cambridge University, 1949

205.11 Beedle--OUTLINE OF STRUCTURAL RESEARCH IN ENGLAND
Dec. 1949

205A.8 Beedle--METAL COLUMNS
June 5, 1948

Weiskopf--FLEXURE OF I-SECTIONS ABOVE THE ELASTIC RANGE
Manuscript, Sept. 19, 1950

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<u>NOTE:</u> Commencing October 1, 1951 all reports from the project will be distributed as "Technical Report No.____". This will be done to avoid the confusion between numbered and lettered reports. The sequence of published "Progress Reports" will be maintained.

NOMENCLATURE AND TERMINOLOGY (REVISED)

Nomenclature

A	Area of cross-section.
B	For a stress-strain diagram, the intercept on the stress axis of the strain-hardening modulus line extended.
b	Flange width.
C	Strain-hardening modulus, $d\sigma/d\epsilon$.
d	Depth of section.
E	Modulus of elasticity.
E_t	Tangent modulus.
F	Force.
f	Shape factor, M_p/M_y, Z/S.
G	Modulus of elasticity in shear.
I	Moment of inertia.
I_e	Moment of inertia of the elastic part of a section.
I_p	Moment of inertia of the plastic part of a section.
L	Total length of a column or a beam.
ΔL	Equivalent length of connection.
M	Moment.
M_h	"Haunch" moment, the moment at intersection of neutral lines of girder and column.
M_p	Full plastic moment, often termed "plastic hinge" moment.
M_{pc}	Ultimate moment of a column modified by compression load.
M_s	Maximum moment of simply supported beam.
M_y	Moment at which flexural yield point is reached.
N	Normal force.
P	Load on a column.
P_{cr}	Critical or collapse load on a column. In the case of axial load alone it is P_E for buckling in the elastic range and approaches P_r in the inelastic range.
P_E	Euler buckling load.
P_r	Reduced or double modulus load.

P_t	Tangent modulus load, the load at which bending of a perfectly straight column may commence.
Q	Static moment of cross-section from outer fibre to neutral axis.
r	Radius of gyration.
S	Section modulus.
s	Core radius.
t	Flange thickness.
W	Concentrated load on a beam.
w	Thickness of the web of an I-section or a plate.
Z	Plastic modulus.
Z_p	Plastic modulus of the plastic part of a section.
ϵ	Strain.
ϵ_s	Strain at beginning of strain-hardening.
ϵ_y	Strain corresponding to σ_y/E.
θ	Rotation
σ	Normal stress, bending stress.
σ_p	Stress at the proportional limit.
σ_r	Residual stress function.
σ_{rc}	Compressive residual stress.
σ_{rt}	Tensile residual stress.
σ_y	Lower yield point stress.
σ_{uy}	Upper yield point stress.
τ	Shear stress.
ϕ	Curvature, rotation per unit length.
ϕ_A	(For connections). Rotation determined on the basis of equivalent length.

Definition of Terms -- Terminology

Buckling Load: Maximum load carried by a member which fails due to instability. P_E or P_r.

Idealized stress-strain diagram:

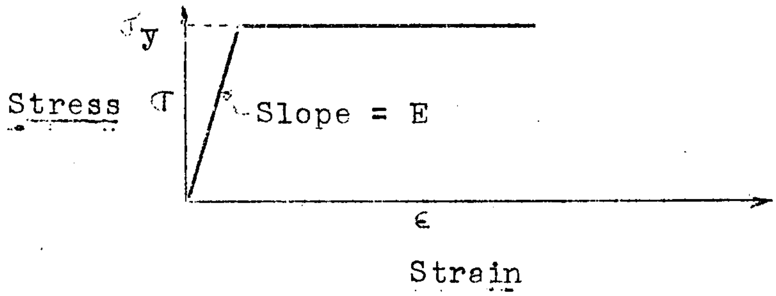

Fig. 1

Initial Yield (Load): A computed load or moment at which the maximum stress reaches the yield point.

Local Yield: Plastic flow due to local effects such as residual stress or stress concentration.

Lüders Lines: (or "Lueders lines") wedges or planes of yielding which form in mild steel at the yield point.

Modified Plastic Hinge Moment: M_{pc}

Perfectly Plastic (material): The ability of a material to deform without an increase in stress after having reached σ_y.

Plastic Hinge Moment: M_p (Plastic moment)

Plastic Modulus: Z, the static moment of the entire cross-section about its neutral axis.

Shape Factor: $Z/S = f$.

Stiffness (rigidity): (Interchangeable) Moment divided by total rotation over an equivalent length, or moment divided by average unit rotation.

Tangent Modulus Load: The load at which a perfectly straight axially loaded column may start to bend. The maximum or critical load is usually only slightly higher.

Yield Lines: Flaking of mill scale following formation of Lüder's lines as revealed by whitewash.

Yield Strength: Load or moment at which a significant amount of yielding has occurred - as indicated by available criteria.

CHART OF COLUMN PROBLEMS (REVISED)

(Revision to preliminary form contained in Progress Report K)

I. MODE OF FAILURE

A. Buckling Failure (elastic and inelastic)

1. Bending deflection (integral collapse)
2. Combined bending and twist (lateral buckling)
3. Local Buckling

B. Plastic "Hinge" Action

1. Instability due to formation of plastic zones or "hinges"
2. Secondary collapse due to rotation at plastic hinges

C. Brittle Fracture

D. Failure Due to Repeated Loading in the Plastic Range (Shakedown)

II. INFLUENCING FACTORS

A. Type of Member

1. Cross-sectional form (prismatic members)
2. Slenderness ratio
3. Non-uniform cross-sections
4. Initial eccentricities and initial curvature
5. Action of splices

B. Type of Structure (columns in trusses, rigid frames)

C. Boundary Conditions (load and restraint)

1. Type and location of loads

   a. End forces and moments (3 coordinate axes)
   b. Intermediate loads: lateral and longitudinal

2. End Restraints (3 coordinate axes) and supports
3. Lateral Support
4. Encasement
5. Action of End Connections

D. Mechanical Properties of Material

1. Type of Material
2. Variation in material properties throughout member
3. Effect of fabrication processes on material properties (strain-hardening, aging, brittleness)
4. Residual stress due to fabrication process

E. Time Effects

1. Blast loading
2. Fatigue